EXPERIMENTAL MULTI-LEVEL SEISMIC PERFORMANCE ASSESSMENT OF RC FRAME BUILDING DESIGNED FOR DAMAGE AVOIDANCE

Brendon A. Bradley

Dept. of Civil Engineering, University of Canterbury, New Zealand

Rajesh P. Dhakal, John B. Mander

Dept. of Civil Engineering, University of Canterbury, New Zealand

Abstract

This paper investigates the experimental application of a newly-developed Multi-level Seismic Performance Assessment (MSPA) methodology. Firstly, MSPA enables each seismic performance requirement to be verified using a single earthquake record, and secondly by applying the earthquake records for the different performance criteria in an increasing severity order, all seismic performance requirements can be verified using a single specimen. This idea is applied in this paper in order to verify the three seismic performance requirements (i.e. immediate occupancy in a moderate earthquake, reparability after a rare earthquake, and life safety in a rare earthquake) for a reinforced concrete frame building designed for damage avoidance. A 3D beam-column joint subassembly designed for damage avoidance is subjected to biaxial quasi-earthquake displacement (QED) tests using earthquake records identified to represent the hazard levels corresponding to the three seismic performance requirements. The subassembly performed well reaching drifts up to 4.7% during the final stage. Although some repairable damage in the form of minor spalling, cracking, yielding of the post-tensioned tendons and energy dissipaters occurred, the subassembly did not collapse. The subassembly met each of the three seismic design criteria, verifying the superior behavior of structures constructed according to the emerging damage avoidance design (DAD).

Keywords: Multi-level Seismic Performance Assessment (MSPA), Incremental Dynamic Analysis (IDA), Damage Avoidance Design (DAD), Quasi-Earthquake Displacement (QED) Test.

INTRODUCTION

Performance Based Earthquake Engineering (PBEE) requires that the performance of structures is evaluated over a range of seismic intensity, as opposed to historical prescriptive frameworks which generally require only a single evaluation at the Design Basis Earthquake (DBE) intensity level. Normally, three different performance requirements need to be verified; the functionality of the structure should not be disrupted by a moderate earthquake (immediate occupancy), the structure should possibly be repaired and reused with a short disruption of functionality after a rare earthquake, and the structure must not collapse after a rare earthquake (life safety). Each of these performance requirements has to be verified either experimentally or analytically, which can be performed by either an equivalent static analysis method (for simple structures) or a nonlinear time-history analysis (needed for irregular structures). Acknowledging that structural response to different earthquake records, despite being scaled to the same intensity measure, will be markedly different, design codes require at least three earthquake records (preferably many more) to be used to get an appropriate confidence on the capacity of the designed structure. Hence, many time-history analyses are required to verify all seismic design criteria. The problem becomes even more severe if the verification is performed experimentally; many specimens should be prepared and subjected to different earthquake records representing the three hazard levels, which is extremely demanding in terms of resources.

In order to make the experimental seismic performance assessment less resource consuming, two improvements to the traditional approach are proposed. The first is to identify earthquake records that give the desired level of confidence in the structural response at the intensity measure corresponding to the three seismic performance requirements. This enables each criterion to be verified using only one earthquake record; i.e. only one specimen will be enough to verify each requirement. Dhakal et al. (2006) have established a method of identifying ‘critical’ ground motion records for the seismic performance assessment of structures. The second improvement is to apply the three pre-identified ground motion records in order of increasing seismic demand, thus allowing a single experimental specimen to be used for structural assessment at all performance levels. This improved method of pre-identifying the most appropriate earthquake records for each seismic design requirement and applying them in sequence of increasing severity to a single specimen is termed as Multi-level Seismic Performance Assessment (MSPA).

This paper applies the MSPA methodology to assess seismic performance of a reinforced concrete frame designed according to the Damage Avoidance Design (DAD) philosophy. According to DAD, a moment resisting building frame is designed as an assembly of precast beams and columns, in which the displacement demand of the frame is accommodated by rocking of the specially designed beam-column joint interfaces and the rest of the members/structure remains essentially elastic, thereby avoiding any significant damage in the members. The strength is obtained from unbonded post-tensioned tendons passing through the beams and the required energy dissipation is obtained from external dampers/dissipaters attached across the beam-column rocking interfaces. A three-dimensional beam column joint subassembly from the building frame is tested against the three earthquake records applied in sequence in quasi-earthquake displacement (QED) tests. QED tests are similar to pseudo-dynamic (PD) tests; the only difference being that the displacement history to be applied in QED tests are pre-obtained from a separate nonlinear time-history dynamic analysis, whereas in PD tests the displacement time history is computed in real time using a coupled experimental-analytical system. See Dutta et al. (1999) for full details of QED testing.

When using this experimental MSPA procedure, seismic performance assessment of the structure can be verified through the test of a single specimen, therefore drastically reducing laboratory preparation time and materials. The carryover experimental damage from previous records, which is overlooked in this methodology, is unlikely to be significant going from performance level 1 (elastic response for immediate occupancy) to level 2, but might have some influence going from performance level 2 (which is likely to induce some inelastic response from the structure) to level 3 (collapse prevention for life safety). However, this oversight is justified because of the simplicity of the methodology and the amount of resources saved compared to those required if three different specimens were required to be tested to verify the three performance levels.

EXPERIMENTAL INVESTIGATION

Specimen details/design

As shown in Figures 1a, the 3D beam-column joint specimen to be tested is an exterior joint on the second floor of a ten-storey reinforced concrete frame building. The original 3 x 3 bays building frame was designed according to New Zealand Standard (NZS3101, 1995) for intermediate soil in Christchurch, New Zealand. Keeping all other design variables constant, the same building was designed and detailed according to damage avoidance principles, resulting in precast and prestressed beam and columns being connected to each other at steel armoured rocking interfaces. Further design details for the prototype building are presented by Robertson (2005). The 80% scaled 3-dimensional beam-column subassembly consisted of two beams in one direction (assumed as the earthquake direction), and one beam in the orthogonal direction (which supports the one-way precast concrete flooring units; i.e. gravity direction). All three beams in the subassembly were one-half the bay length and the column one-half the interstorey height of the prototype building; this assumes that the point of inflexion occurs at the beam and column midpoints respectively. A supplemental external damping system was used to provide additional energy dissipation.

Reinforcing layouts of the subassembly are given in Figures 1b and 1c. All beams were 4 m long and had cross-sectional dimensions of 400 mm wide and 560 mm deep, while the column was 700 mm square and had a height of 3.2 m. The beams were prestressed with two 26.5 mm high strength high alloy (Dywidag™) threadbars. A straight tendon profile with a diagonal connecting fuse rod was designed for the seismic beams, while a draped tendon profile was adopted for the “gravity” beam to represent balanced dead load. To protect beam ends from large concentrated forces occurring during rocking behaviour, two 100 mm x 100 mm x 12 mm steel angles were cast at the top and bottom edges of the beams at the connection. Within the mid section of the beams, only minimal transverse steel was used, thus the maximum allowable stirrup spacing of 250 mm centre-to-centre was adopted. Closely spaced (100 mm centre-to-centre) stirrups were placed at beam ends to help transfer large rocking and post-tensioning forces.

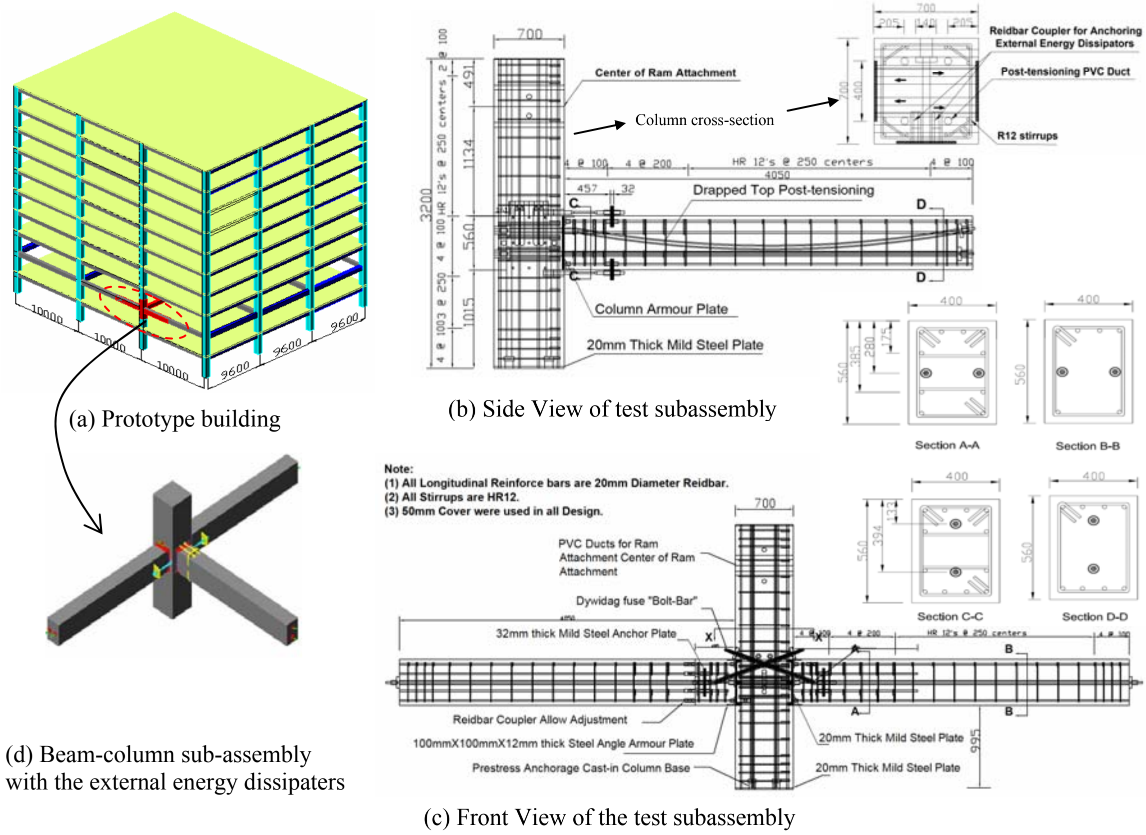


Figure 1: Prototype Building structure and Reinforcement details of subassembly.

Four tapered shear keys were used at beam-column interfaces to carry shear forces, and provide torsional restraint. Tension-compression dissipaters were connected across the interface by screwing to the column at one side and to the steel plates cast into the beams at the other side. Further details of the experimental model of the structure can be found elsewhere (Li, 2006).

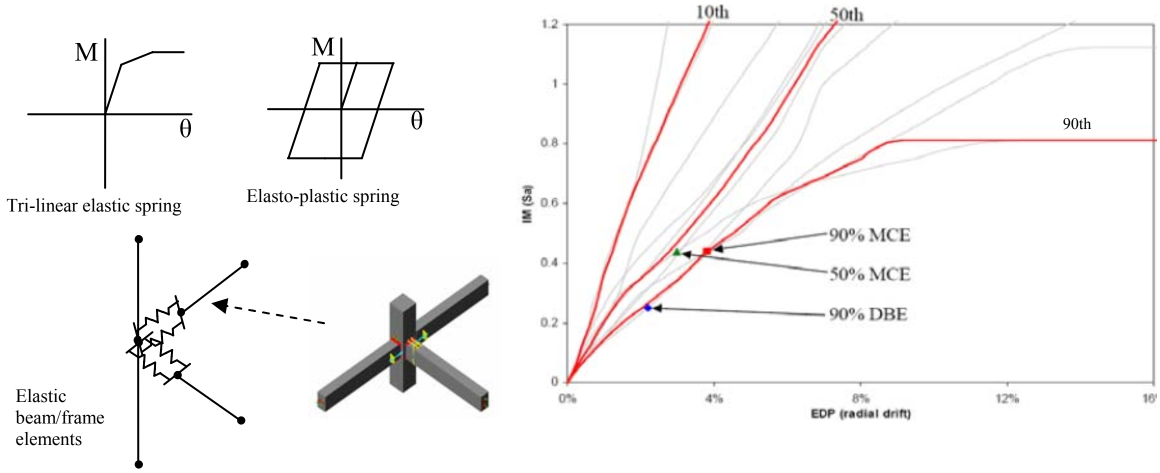
Loading Protocol

In order to perform MSPA, the earthquake records to be used must be pre-identified. Dhakal et al. (2006) have proposed a methodology comprising Incremental Dynamic Analysis (IDA) using several earthquake records and probabilistically processing of the IDA results to identify records giving the desired level of confidence at different intensity levels. IDA (Vamvatsikos and Cornell, 2002) involves conducting non-linear dynamic analyses of a computational model of a structure subjected to a suite of earthquake ground motion records scaled to different intensity measures (IM). For each analysis, an engineering demand parameter (EDP) is monitored, thus producing IDA curves; i.e. plots of IM vs. EDP.

For this study, two suites of 20 earthquake records were used; a ‘near-source’ suite and ‘medium source’ suite, to compare the response of different source mechanisms. Each of the records in the suites represented one of the two horizontal components recorded in fault-normal and fault-parallel components. The components were then combined, rotated 45 degrees and resolved into two directions. This allows the records to be used for both bi-directional and uni-directional computation.

In order to perform IDA, an analytical model of the beam-column joint subassembly was conceptualized using Ruaumoko3D (Carr, 2004). The beams and columns were represented using Giberson beam frame elements, with an elastic hysteresis; which is in line with the expectation of DAD philosophy that all inelastic and non-linear behaviour occurs at rocking joints. The behavior of the rocking joint was described using two springs of zero length in parallel (Figure 2a). The first spring represented the behavior of the unbonded post-tensioning tendon and had a tri-linear elastic hysteresis to represent pre-rocking, gap-opening, and tendon yielding phases, respectively. The second spring was elastoplastic and represented the supplemental energy dissipation system. The parameters of the springs representing the rocking joints were calibrated based on the results of preliminary quasi-static tests (Li, 2006).

A full scale computational model of the 10-storey prototype building was developed. IDA was conducted using the two suites of ground motion records. The IM selected was the 5% damped spectral acceleration at the fundamental time period of the model, which was 1.6 seconds, i.e. IM=S_A (T_1, 5%). Assuming that the fundamental period of the structure T_1 lies in the constant velocity range of the design acceleration spectra, the spectral acceleration can be related to the PGA by S_a = PGA/T_1. Hence, the spectral accelerations for the Design Basis Earthquake (DBE) with 10% probability of exceedence in 50 years (PGA=0.4g) and Maximum Credible Earthquake (MCE) with 2% probability of exceedence in 50 years (PGA=0.72g) are 0.25g and 0.45g, respectively (See Figure 2b).



(a) Computational Modelling

(b) Incremental Dynamic Analysis curves

Figure 2: Modeling and IDA results for earthquake identification

These analyses resulted in a matrix of data points on an EDP (i.e. θ_{max}) vs. IM (i.e. S_a) plot, which were connected to obtain the IDA curve for all earthquake records in the suites. Thus obtained IDA curves for the near source suite of earthquakes are shown in Figure 2b. From the generated IDA curves the structural behavior is assessed at three different levels of performance. The first performance criterion is that at the DBE intensity level, there should be a high probability (assumed 90%) that the structure will suffer no more than minor damage, to ensure immediate occupancy. The second criterion is that at the MCE intensity level there should be a moderate probability (assumed 50%) that structural damage will be repairable. Finally, the third criterion is that at the MCE intensity level there should be a high probability (assumed 90%) that global collapse of the structure will not occur, to ensure life-safety. Hence, 3 records were selected from each of the two suites that are closest to the 90^{th} percentile response at the DBE intensity level (i.e. S_a = 0.25g) and the 50^{th} and 90^{th} percentile responses at the MCE intensity level (i.e. S_a = 0.45g). The identification of these three ground motion records from the near source suite are shown in Figure 2b, and the seismological details of the three earthquakes for the two suites are given in Table 1. These earthquakes were used in MSPA of the building through QED test of the specimen.

For the QED tests, displacement-history at the loading point (top of the column) was required. Nonlinear dynamic time-history analyses were carried out using these critical earthquakes applied to the 10-storey building model, and the displacement response of the nodes at the centre of the columns on the second and the first floor were plotted. As these nodes represented the top and bottom of the column in the subassembly to be tested, the differential displacement-time history of the two nodes gave the interstorey displacement-time history for the beam-column subassembly. Thus obtained bi-directional inter-story drift histories were applied to the top of the column through two hydraulic actuators. Gravity load was applied at the top of the gravity beam to simulate the load coming from the one-way flooring system. The details of the loading and measurement systems can be found elsewhere (Li, 2006).

Table 1 Details of the earthquake records from the two suites identified to represent the three different hazard levels

ID	Event	Year	Station	M[*1]	R[*3] (km)	Component	S_A[*4] (g)	Source	Hazard level	S_A[*5] (g)	Peak Drift (%)
EQ7	Landers	1992	Barstow	7.3	36	EW	0.18	medium	90% DBE	0.25	1.2
						NS	0.093			0.13	
EQ1	Imperial Valley	1940	El Centro	6.9	10	EW	0.136	medium	50% MCE	0.43	1.8
						NS	0.041			0.13	
EQ11	Loma Prieta	1989	Gilroy	7.0	12	EW	0.17	medium	90% MCE	0.42	4.7
						NS	0.16			0.41	
EQ40	Palos Verdes	-	Simulated	7.1	1.5	EW	1.37	near	90% DBE	0.25	2.1
						NS	0.69			0.22	
EQ30	Tabas	1974	-	7.4	1.2	EW	0.56	near	50% MCE	0.42	2.8
						NS	0.50			0.39	
EQ24	Loma Prieta	1989	-	7.0	3.5	EW	1.31	near	90% MCE	0.42	3.8
						NS	0.55			0.18	

[1] Component, [2] Moment Magnitudes, [3] Closest Distances to Fault Rupture, [4] Unadjusted S_A, [5] Scaled S_A.

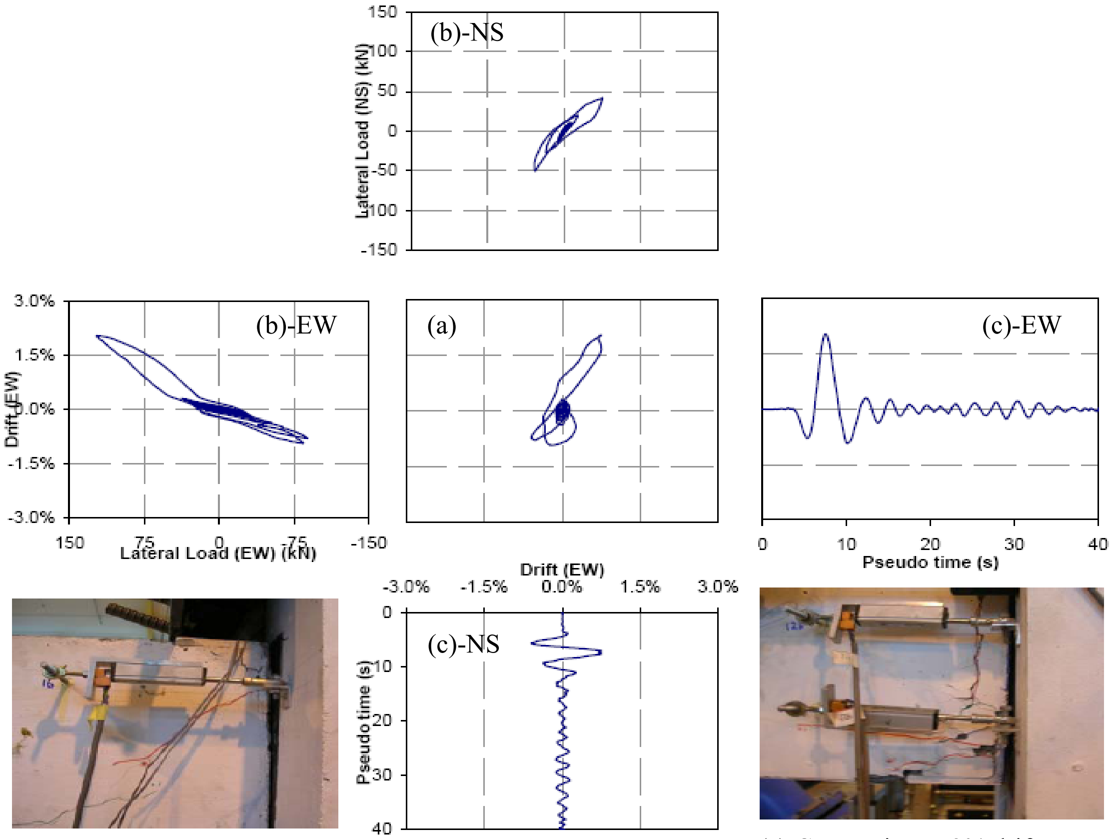
EXPERIMENTAL RESULTS

The final column of Table 1 shows the peak drift responses for each of the six QED tests conducted. The drift demands for the six records obtained from nonlinear time-history dynamic analysis during the IDA also showed similar hierarchy. According to the concept of MSPA, the six records were applied to the subassembly in the order of increasing drift demand. Here, experimental results are presented in detail only for the larger of the two responses at each of the performance assessment levels.

Response to the 90th percentile DBE

The near source ground motion causing the 90th percentile response at DBE was EQ40 which had a peak radial drift of 2.13%. Figure 3c shows the input drifts applied to the subassembly, while Figure 3a shows the drift orbit of the top of the column. Figures 3d and 3e show gap opening on the east and west seismic beams respectively at a drift of 2.13%. Note that quasi-static tests had been conducted on these specimens (Li, 1006) whose results were used to calibrate the analytical models as mentioned earlier. Moreover, the displacement time-histories corresponding to EQ7 (90th percentile DBE medium source record) had been applied to the subassembly prior to the QED test using EQ40. During these tests, some cracks had emerged in the beam near the rocking interface. Consequently, some cracks in the top-half depth and minor spalling in the inside of the bottom steel angle can be seen in Figure 3c.

Due to the nature of near source records, the input drifts are comprised primarily of one peak occurring simultaneously in both orthogonal directions. The peak drifts on this cycle were 1.95% and 0.64% in EW and NS directions, respectively. The corresponding lateral forces for the EW and NS directions were 118.9 kN and 38.1 kN. On the maximum drift cycle, the supplemental energy dissipators on the seismic beams yielded, however no noticeable buckling occurred during unloading indicating only partial yield of the dissipater, which can also be guessed by observing the moderate energy dissipation occurring during the peak drift cycle (Figure 3b). Overall, the specimen performed satisfactorily, and it was obvious that the specimen did not need any repair to restore the functionality after this test.



(d) Connection at 2% drift

(e) Connection at 2% drift

Figure 3: Experimental Results for the 90% DBE near source ground motion (EQ40).

Response to the 50th percentile MCE

The near source ground motion causing the 50th percentile response at MCE was EQ30 which had a peak radial drift of 2.84%. Figure 4c shows the input drifts in the two directions, while Figure 4a shows the drift orbit of the top of the column. The peak drifts were 2.72% in the EW direction and 1.77% in the NS direction. The corresponding lateral forces in EW and NS directions were 149.4 kN and 77.5 kN, respectively. As can be seen in Figure 4b, the drifts are not exactly zero when the lateral load was fully unloaded. It was similar during the previous QED tests with EQ7 and EQ40 (Figure 3b). This apparent lack of re-centering capability in the lateral load vs. drift plot was due to the large friction forces that occurred between the Dywidag bolt bar (used for connecting the straight tendon and the inclined "fuse" bar to form the post-tensioning system) and the enclosing PVC duct. The bent coupler profile also resulted in the occurrence of "shuddering" noises when friction between the duct and bolt bar was overcome and slip occurred.

The lateral load vs. drift hysteretic responses in Figure 4b (especially in the NS direction) show good energy dissipation on the peak drift cycles. Upon re-centering of the gravity beam from the maximum drift cycles in either direction, buckling of supplemental energy dissipators occurred (Figure 4f). The lateral load vs. drift response in the EW direction (Figure 4b) shows very good energy dissipation during peak drift cycles occurring at 18 and 21 seconds in the positive and negative directions, respectively. However, in the aftermath of these large drift cycles, energy dissipation at subsequent cycles of lower drift reduced due to buckling of the supplemental energy dissipators.

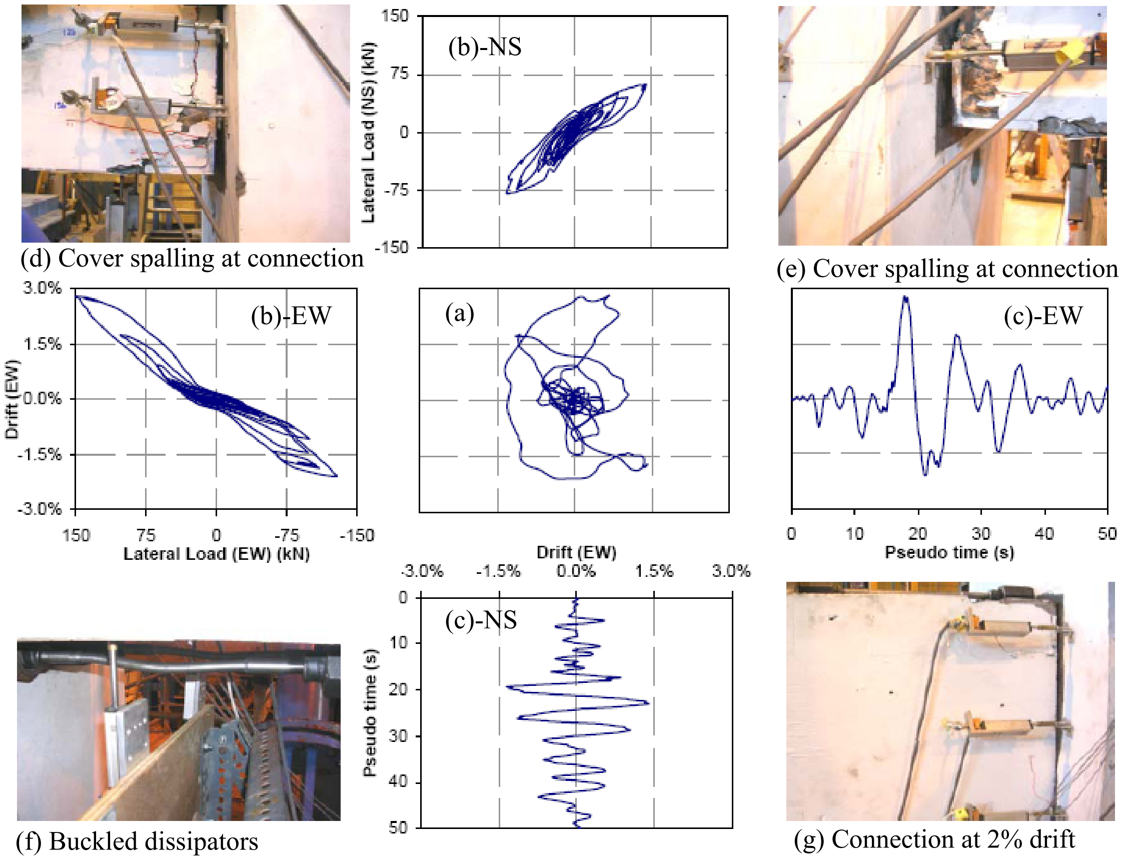


Figure 4: Experimental Results for the 50% MCE near source ground motion (EQ30).

Minor strength degradation is apparent during the two large cycles in the negative EW drift direction (Figure 4b). As the damage in the specimen at this stage was not significant, it was deduced that damage was not contributing much to the strength degradation. The cause of this is believed to be the reduction in efficiency of dissipators due to buckling, which occurred during unloading from the previous large drift cycle in the positive EW direction. Figures 4d and 4e show the lower steel angle on the west seismic beam viewed from the North and South respectively. Previous crushing and spalling caused a reduction in load carrying capacity of concrete around these affected regions, and at the peak drift of 2.84% some spalled concrete fell off the beam. After the test, it was apparent that the external energy dissipater needed to be replaced. This could easily be managed without causing any disruption to the building occupants.

Response to the 90th percentile MCE

The medium source ground motion generating the 90% percentile response at the MCE intensity level was EQ11 which had a peak radial drift of 4.69%. The QED test results for EQ11 are shown in various forms in Figure 5. Figure 5c shows the input drifts in the two directions, while Figure 5a shows the drift orbit of the top of the column. Peak drifts were 4.68% in the EW direction and 3.49% in the NS direction. The corresponding lateral forces in EW and NS directions were 171.5 kN and 77.8 kN, respectively. Note that, unlike the five previous tests which had drift demands in the NS (gravity frame) direction less than 40% of that in the EW (seismic frame direction), the NS drift demand in this test was more than 70% of the EW drift demand.

The lateral load vs. drift response in the NS direction is shown in Figure 5b, which shows good energy dissipation during the peak drift cycles in both positive and negative directions. No significant damage occurred in the gravity beam other than very minor spalling at corners of the connection and a few hairline flexural cracks. The EW lateral load vs. drift hysteresis is also shown in Figure 5b. It can be seen that good energy dissipation was provided by the supplemental energy dissipation system. The peak drift of 4.68% in the EW direction caused additional damage to the beam in the form of concrete spalling/crushing at rocking edges as shown in Figure 5e. Large drifts in the EW direction also led to slip in the connection between the supplemental energy dissipators and column, which occurred after unloading from the largest positive drift of 4.25%. It can be seen that in the aftermath of large drift cycles (in both NS and EW directions), energy dissipation for smaller drift cycles is relatively small (Figure 5b). This occurred because upon re-centering from large drift cycles buckling of dissipators occurred (Figure 5g).

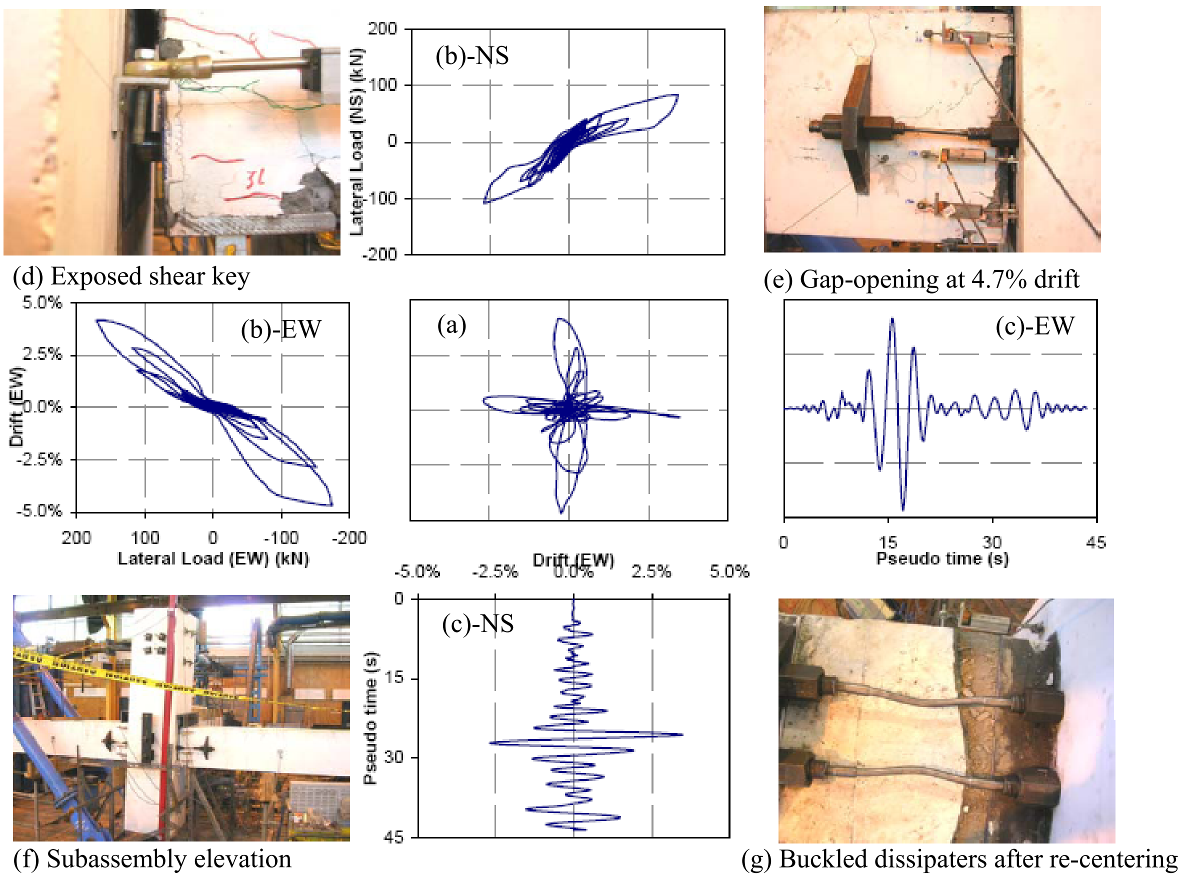


Figure 5: Experimental Results for the 90%s MCE medium source ground motion (EQ11).

As can be observed from the hysteresis loop corresponding to the maximum EW drift of 4.68% in Figure 5b, a flat plateau exists near the peak and also significantly higher energy dissipation is apparent in the hysteresis loop. This is because of the yielding of all four 'fuse' bars (thinner inclined segments of the post-tensioned tendons) in the EW beams. Despite this yielding the subassembly's re-centering capability did not deteriorate, with a residual drift of only 0.2% (6 mm). However, inelastic strain caused by yielding of the tendons did reduce post-tension forces, and as a result the bi-linear stiffness of the subassembly reduced during subsequent cycles. The test indicated that the specimen needed to be repaired and the post-tensioned tendons may need to be tightened to overcome the loss of prestress level due to yielding.

DISCUSSION: EXPECTED PERFORMANCE OF THE BUILDING WITH RESPECT TO THE SEISMIC DESIGN CRITERIA

As previously mentioned, earthquake records representing different levels of seismic hazard were applied in sequence to the same specimen in these tests to perform multi-level seismic performance assessment (MSPA). The test results can now be interpreted in terms of MSPA and the seismic performance of the building at three levels of seismic demand can be extrapolated based on these test results.

The first level of seismic performance is to review the response of the structure to the designed level of earthquake: i.e. DBE with intensity of 0.4g PGA. At this level of seismic hazard, one needs to have high confidence that no significant damage needing major repair will occur (so that the structure keeps functioning without any disruption after the earthquake). In fact, there exists, in many cases, one more level of performance requirement prior to this; that is the structure should have no damage at all during frequent earthquakes. These frequent earthquakes have intensities significantly less than that of DBE; and this preliminary seismic performance requirement has not been incorporated into this study. To ensure that the structure will suffer no damage due to the DBE, the structure needs to be tested against an earthquake causing larger response and severer damage at the DBE intensity level than any other earthquakes scaled to the same intensity level. EQ40 was selected for this purpose because it generated the 90th percentile response at DBE level among the 40 earthquakes in the two suites. During the 90th percentile DBE test, no new cracks were observed in the members, although minor cracks formed during the previous quasi-static and QED tests were present. Although the external dissipaters yielded, buckling was not evident and replacement of dissipators was not required. The subassembly therefore satisfied the first requirement of the performance based seismic design.

The second and third levels of seismic performance assessment refer to structural response at rare earthquakes; i.e. MCE with intensity of PGA = 0.72g for the location assumed in this study. The second performance requirement indicates that there must be moderate confidence that the induced damage will be repairable after an earthquake of MCE intensity level. This requirement ensures that a fair possibility exists that the structure will not need to be dismantled and can be repaired to regain its functionality even after rare and large earthquakes. To ensure that the structure will only suffer moderate damage requiring repair work due to the maximum considered level of seismic hazard, the structure needs to be tested against an earthquake giving moderate confidence (assumed 50% in this study) of representing the MCE intensity level. EQ30 generated the 50th percentile response at MCE level among the earthquakes selected, and had a peak drift of 2.84%. QED tests using this record (i.e. EQ30) showed buckling of supplemental energy dissipators upon re-centering, which caused low energy dissipation during subsequent drift cycles. No strength degradation of beams or column were observed, however buckling of supplemental energy dissipators caused minor global strength degradation. Some spalled concrete fell off the specimen at the peak drift. In order to restore the specimen following this 50th percentile MCE test, patching of spalled concrete around the steel angles and replacement of external dissipators would be required. Therefore, it can be concluded that the structural performance was adequate to satisfy the second seismic performance requirement.

The third and final seismic performance requirement demands that there must be high confidence that the structure will not collapse in a rare earthquake; i.e. represented by MCE with a 2% probability in 50 years. This requirement is to ensure life-safety of people even in a maximum considered earthquake. To ensure that the structure will only suffer, at most, severe damage requiring major repair work, but not collapse due to the maximum considered level of seismic hazard, the structure needs to be tested against an earthquake giving high confidence (assumed 90% in this study) of representing the MCE intensity level. EQ11 generated the 90th percentile response at the MCE intensity level and had a peak drift of 4.69%. QED test using EQ11 showed spalling of concrete in the seismic beams at the beam-column connections, severe buckling of the supplemental energy dissipators, yielding of post-tensioning tendons, and minor flexural cracks in the beams. Note that these damages are also contributed by the carryover effect from the quasi-static test and QED tests with five previous records conducted before this test. No significant strength degradation however was observed in the lateral load vs. drift curve obtained from this test. Yielding of the post-tensioning “fuse” bars did not affect the re-centering capability of the subassembly. Repair work required would be replacement of external dissipators, patching of spalled concrete and re-stressing of yielded tendons. Most importantly, the specimen did not collapse. It can hence be concluded that the subassembly easily satisfied the third and final performance based seismic design criteria.

CONCLUSIONS

Based on the experimental results the following conclusions can be drawn:

1. In order to apply multi-level seismic performance assessment (MSPA) methodology to a moment resisting RC frame building designed according to Damage Avoidance Design (DAD) philosophy, quasi earthquake displacement (QED) tests were conducted on a 3D beam-column joint subassembly from a critical part of the building frame.
2. Following the MSPA methodology, earthquake records representing different levels of seismic hazard were pre-identifying and applied in sequence of increasing demand to the same specimen. The MSPA enabled all three seismic performance requirements to be verified by testing a single specimen.
3. One can be some 90% confident that the 10-storey DAD building (the subassembly is a critical part of this building) will survive a design basis earthquake (DBE) without needing a major repair and without causing any downtime.
4. One can be at least 50% confident that the DAD building can survive a maximum considered earthquake (MCE) without needing to dismantle it. After some downtime needed to repair the incurred damage, the building could be reused after an MCE.
5. One can be at least 90% confident that the DAD building will not collapse in an MCE thereby ensuring the most important requirement of life safety.

REFERENCES

Bradley, B.A., Dhakal, R.P., Mander, J.B. 2006 Dependency of current Incremental Dynamic Analysis to source mechanisms of selected records, 19th Biennial Conference on the Mechanics of Structures and Materials (ACMSM19), Christchurch, New Zealand. (In Press)

Carr A.J., 2004, Ruaumoko3D: Inelastic Dynamic Computer Program, Computer Program Library, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Dhakal, R.P., Mander, J.B., Mashiko, N., 2006, Identification of critical ground motions for seismic performance assessment of structures, Earthquake Engineering and Structural Dynamics, Vol. 35, Issue 8, pp. 989-1008.

Dutta, A., Mander, J.B., Kokorina, T. 1999, Retrofit for Control and Reparability of Damage, Earthquake Spectra, v 15, n 4, Nov, p 657-679

Li. L, 2006, Further Experiments on Damage Avoidance design of Beam-to-column joints, ME Thesis, Dept. of Civil Engineering, University of Canterbury, Christchurch New Zealand

Robertson K., 2005, Probabilistic Seismic Design and Assessment Methodologies for the New Generation of Damage Resistant Structures, ME Thesis, Dept. of Civil Engineering, University of Canterbury, Christchurch New Zealand

Standards New Zealand 1995, NZS 3101: Part 1: 1995: Concrete Structures Standard, Standards New Zealand, Wellington, New Zealand.

Vamvatsikos D., Cornell, C.A., 2002, Incremental Dynamic Analysis, Earthquake Engineering and Structural Dynamics, Vol.31, pp 491–514.